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DISCUSSION OF ICE PRESSURE AGAINST DAMS: STUDIES OF THE EFFECTS OF TEMPERATURE VARIATIONS PROCEEDINGS-SEPARATE NO. 160

G. E. MONFORE[9].—The experimental approach used by Mr. Löfquist in investigating ice pressure indicates thoughtful planning and careful measurements. This approach, which involved the use of what might be called a laboratory ice sheet 20 in. in diameter and 24 in. thick, is intermediate to the methods used by the Bureau of Reclamation (USBR), United States Department of the Interior. The investigations conducted by the USBR[10] consisted of field measurements of pressure and temperatures at several reservoirs and laboratory measurements on 4-in.-by-4-in. ice cylinders. The investigations by Mr. Löfquist and by the USBR are thus entirely independent and so provide a valuable check on each other.

Solar radiation, which is an important factor in the temperature changes in an ice sheet in the field, is difficult to simulate in a laboratory ice sheet. The flow of energy into the laboratory ice sheet must come mainly from the air that is in contact with the ice surface, whereas in the field ice sheet, energy comes not only from the air but by absorption of solar radiation. The limited USBR experiments indicated that about 35% of the solar energy is absorbed after passing through 1 in. of ice and 60% is absorbed after passing through 25 in. of ice.[6] Thus, much radiant energy is transformed to heat near the ice surface, and additional heat is generated lower in the ice sheet. It follows that a given rate of air temperature rise in the laboratory will produce a lower rate of increase in the ice temperature of the laboratory ice sheet than that produced in the field ice sheet exposed to solar radiation in addition to the same air temperature rise. Variation of ice temperature with depth in the laboratory ice sheet will probably differ from that in field ice sheets.

It was interesting to note that the author found a marked reduction in the maximum pressure when there were contraction cracks in the laboratory ice sheet. The USBR observed the same phenomenon in thier field measurements. Apparently under some field conditions, the contraction cracks fill with water which then freezes, but under other conditions the cracks remain open. The open cracks prevent the development of high pressures. A better understanding of the factors that allow open cracks to persist in field ice sheets might aid in fixing an upper limit for ice thrust.

The pressure curves obtained by Mr. Löfquist from the laboratory ice sheet and the curves that the USBR obtained from field ice sheets and from small laboratory samples are all of a similar shape. As the ice temperature increased, the ice pressure increased to a maximum value and then decreased.

[9] Research Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

[10] "Ice Pressure Against Dams: Experimental Investigations by the Bureau of Reclamation," by G. E. Monfore, *Proceedings-Separate No. 162*, ASCE, December, 1952.

[6] "Ice Pressure Measurements at Eleven Mile Canon Reservoir During January, 1949," by G. E. Monfore, *Report No. SP-21*, Structural Research Lab., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., April 28, 1949.

The temperature at which the maximum pressure occurs may be considerably less than the melting point of ice. These pressure curves indicate that the maximum pressure that can be developed in an ice sheet is limited by plastic flow of the ice.

The data from Fig. 5 may be compared with the results of the USBR laboratory study as shown in curves published in a companion paper.[11] The data from Gage No. 5, Fig. 5(*b*), were used for comparison because the ice temperature in this case increased almost linearly until the maximum pressure was reached. Linear rates of ice temperature rise were used in the USBR laboratory study. According to Fig. 5(*b*), the initial ice temperature was $-2°$ F, and the ice temperature increased from $-2°$ F to 24° F in 17 hr, a rate of rise of 1.5° F per hr. The maximum pressure registered by Gage No. 5 was 95 lb per sq in.

[11] "Ice Pressure Against Dams: Experimental Investigations by the Bureau of Reclamation," by G. E. Monfore, *Proceedings-Separate No. 162*, ASCE, December, 1952, p. 11, Fig. 9.

The maximum pressure obtained in the USBR laboratory study for similar temperature conditions may be read from the previously mentioned USBR curves.[11] An ice temperature rise of 1.5° F per hr from an initial temperature of $-2°$ F resulted in a maximum pressure of 125 lb per sq in. which occurred in from 5 hr to 6 hr.

In the USBR laboratory study, the ice samples were maintained at a constant length, whereas in the author's work the ice sheet could expand a small amount because of the thermal expansion of the concrete vessel and the deformation of the vessel under load. This limited expansion of the laboratory ice sheet would decrease the magnitude of the maximum pressure and increase the time required for the maximum pressure to develop. Mr. Löfquist estimates that if there were no thermal expansion of the vessel, the maximum pressure would have been about 25% greater. This corrected maximum pressure would be $95 + \frac{95}{4} = 120$ lb per sq in., which is in excellent agreement with the pressure of 125 lb per sq in. measured in the USBR laboratory study.

Edwin Rose.[12]—The results of the tests performed by Mr. Löfquist represent a definite contribution toward obtaining a better understanding of the ice-pressure problem. In a paper published in 1947,[4] the writer presented a basis for estimating ice pressures against hydraulic structures by the application of the ice temperature-pressure relationship established by Messrs. Brown and Clarke in 1932.[3] The writer stated that more data, both from the laboratory and from the field, were needed to verify this relationship and to make it possible to prepare more reliable design criteria. Additional data have been presented by Mr. Löfquist, some of which differ considerably from the data presented by Messrs. Brown and Clarke.

[12] Structural Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

[4] "Thrust Exerted by Expanding Ice Sheet," by Edwin Rose, *Transactions*, ASCE, Vol. 112, 1947, p. 871.

[3] "Ice Thrust in Connection with Hydro-Electric Plant Design," by Ernest Brown and George C. Clarke, *The Engineering Journal*, January, 1932.

The measured temperatures shown in Fig. 4 can be checked by the method of computation presented in the writer's paper. Applying the relationships obtained by Messrs. Brown and Clarke to these temperature changes and estimating the effect of restraint by the use of Poisson's ratio equal to 0.365, results in an estimated maximum pressure of approximately 4,000 lb per lin

ft rather than the 13,400 lb per lin ft shown in Fig. 6. Results of the author's experiments show that considerably greater pressure is developed than is indicated by the basic data presented by Messrs. Brown and Clarke. Figs. 5 and 6 reveal a pear-shaped pressure distribution, with the maximum intensity reached in the upper parts of the ice sheet and varying to a lower value at the top surface, and to zero pressure at the bottom of the ice sheet. Fig. 6 shows intensities of pressure that build up to a maximum and then undergo a characteristic relief of pressure with the passage of time. This plastic-flow effect presents a more logical behavior for this material than the distribution of ice pressure obtained by Messrs. Brown and Clarke.[13]

[13] "Thrust Exerted by Expanding Ice Sheet," by Edwin Rose, *Transactions*, ASCE, Vol. 112, 1947, pp. 896-898.

It appears that the curve of Fig. 7 yields results that are different from the results obtained by the use of Figs. 3 to 6, inclusive. This seems to be caused by the fact that relief of pressure with time cannot be evaluated from the type of curve shown in Fig. 7.

For thicker ice sheets a lower rate of pressure increase occurs, but a maximum pressure (per linear foot) may not be reached when the ice sheet is 2-ft thick. Some pressure increase will probably develop in thicker ice sheets for the same air temperature and other conditions. A sample computation of curves A and B in Fig. 8 would aid in illustrating the method by which values on these curves were determined. It would also indicate whether the application of the mean-temperature concept might lead to pressure values of doubtful accuracy. The writer has stated that the high pressures shown for thick ice sheets were unlikely because these pressures were dependent on a sustained rate of air-temperature rise, and that pressure would be relieved by plastic flow with the passage of time.[14,15,16] These statements were made because a conservative approach was believed justified in the application of the data obtained by Messrs. Brown and Clarke. Mr. Löfquist's experiments, and those of Mr. Monfore,[10] indicate that the pressures determined by Messrs. Brown and Clarke were low, especially in the lower range of ice-temperature rises. Also, their curves did not show the relief of pressure with the elapse of time after the maximum value had been reached, although this characteristic is mentioned in the text of the paper, under the heading, "Prior Investigations".

[14] *Ibid.*, p. 883.
[15] *Ibid.*, p. 897.
[16] *Ibid.*, p. 898.

It would be helpful to have information on the following:

a. Test data for more rapid rates of ice-temperature rise to extend the curve in Fig. 7.

b. The field use of pressure gages in a large ice sheet, especially if this ice were thick.

c. Ice pressures that might develop if the water in the bottom of the freezing chamber were prevented from escaping as the ice froze. This would explain the effect of (1) confinement and (2) size of well or pit in the development of forces caused by a volume expansion as a result of the freezing process. Under what conditions of size and restraint does this force become important?

In reviewing the data shown in Figs. 3 and 5, differences are noted in the depths of the gages. It is assumed that the depths given in Fig. 5 are the correct ones.

The pressures obtained by Mr. Löfquist are higher than the pressures obtained by Messrs. Brown and Clarke. These higher pressures are caused partly by the two-dimensional stress conditions existing in Mr. Löfquist's tests, possible variations in test procedure, and ice crystal structure. However, some further explanation should be made of the differences. Ultimately a new and consistent set of basic data should be set up when such data are well founded.

Thus the problem does not appear entirely solved. Perhaps a precise solution is not possible, but some revision to the writer's curves or some new data should be prepared—coordinating the findings of Messrs. Löfquist and Monfore with the findings of A. D. Hogg[17]—when sound and consistent results are established. The data should be presented in a form to permit quick and easy use by designers of hydraulic structures to aid them in their judgment of the magnitude of ice pressures to be assumed in the design of such structures.

[17] "Ice Pressure Against Dams: Some Investigations in Canada," by A. D. Hogg, *Proceedings-Separate No. 161*, ASCE, September, 1952.

Bertil Löfquist.[18]—Mr. Monfore has shown by an example that the writer's values for the maximum ice pressure are in good agreement with the pressures measured in the USBR laboratory study. The time required to reach this maximum pressure varies, however. The difference may be caused by a variation in the crystalline structure of the ice, but it more probably results from differing strain conditions. In the USBR study, the ice cylinder was restrained in the axial direction; but in the writer's study, the ice cylinder was restrained in the radial directions. The ice pressure is expected to be greater under the latter conditions, but other factors in the tests may compensate for this difference—such as adhesion between the ice and the bearing blocks in the USBR study and cracks in the ice in the writer's study.

[18] Chf. Inspector, Swedish State Power Board, Stockholm, Sweden.

Solar radiation is generally insignificant during the winter in northern Sweden, but it is surely an important factor in the ice pressures in more southerly areas.

The ice pressures measured in the USBR field investigations are, at present, the most reliable. As the values are in good agreement with laboratory tests, they represent a definite source of information concerning ice pressure.

The relation between the ice pressure and ice thickness is a question that needs further investigation. Mr. Rose does not agree that the ice pressure has a maximum at a thickness of approximately 2 ft, and thereafter decreases slightly, or in any case only increases to an insignificant extent (Fig. 8). This conception is a result of computations based on the writer's test and some simplified assumptions as given in the paper. Mathematical treatment of ice-pressure problems is apt to be misleading, however, and the writer will not stress these results. If solar radiation is considered, another relation must exist.

The disagreement between the data in Figs. 3 and 5, to which Mr. Rose refers, is explainable. The "gages" in Fig. 3 are thermocouples, and the "gages" in Fig. 5 are pressure gages. The gage numbers in the two figures do not correspond.

DISCUSSION OF ICE PRESSURE AGAINST DAMS: EXPERIMENTAL INVESTIGATIONS BY THE BUREAU OF RECLAMATION PROCEEDINGS-SEPARATE NO. 162

EDWIN ROSE.[13]—The results of extensive field and laboratory investigations are a worthwhile addition to the information regarding the development of ice pressure caused by the expansion of an ice sheet resulting from temperature changes.

[13] Structural Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

Mr. Monfore has obtained a favorable comparison between ice pressures obtained from direct field measurement and those computed by application of the temperature-pressure relationships established in the laboratory to the corresponding ice temperatures measured in the field.

Although not in a form that permits direct comparisons, the ice-pressure values shown in Figs. 7 to 9 appear to differ from the results of similar tests conducted in 1932 by Ernest Brown, A.M. ASCE, and George C. Clarke,[10] M. ASCE. Figs. 7 and 9 show that for a given rate of ice-temperature rise, the rate of ice-pressure rise is very rapid for 1 hr to 3 hr. A maximum pressure is reached in from 1 hr to 6 hr depending on the initial ice temperatures and the rate of ice-temperature rise. After the maximum value is reached the pressure decreases. During the early period, the rate of pressure increase is greater than that shown by the tests performed by Messrs. Brown and Clarke. In a paper published in 1947[14] the writer computed curves in a form convenient for design purposes based on the ice temperature-pressure relationship established by Messrs. Brown and Clarke. Use of the author's test data, in a similar manner, indicates that greater pressures would result in the case of a low rate of ice-temperature rise and in ice sheets 2 ft or less in thickness. Cases of more rapid temperature rises and for thicker ice sheets indicate that maximum pressures are more nearly comparable.

[10] "Ice Thrust in Connection with Hydro-Electric Plant Design," by Ernest Brown and George C. Clarke, *The Engineering Journal*, January, 1932, pp. 18-25.

[14] "Thrust Exerted by Expanding Ice Sheet," by Edwin Rose, *Transactions*, ASCE, Vol. 112, 1947, pp. 871-900.

Under the heading "Computation of Ice Thrust," Mr. Monfore used the maximum values at the top surface and the 8-in. depth to obtain the total pressure through the ice. Since these pressures did not occur at the same time, this procedure yields too large a maximum value of total pressure. By observation of the curves in Figs. 7 and 9, the author shows that the pressure at a given point increases to a maximum in from 1 hr to 6 hr and then decreases. This general pattern also is shown by the field data on pressures measured with the electric gages. This characteristic property should be used in the computation procedure.

To permit computation of pressures at various depths of the ice sheet at different increments of time, it would be helpful if the curve of Fig. 7 could be extended to show the pressure decrease past the maximum as the ice tempera-

ture continues to rise at a uniform rate. Such pressure values could then be integrated to obtain the total pressure in the ice sheet at a given time, and from this the maximum pressure value could be determined. Bertil Löfquist[15]

[15] "Ice Pressure Against Dams: Studies of the Effects of Temperature Variations," by Bertil Löfquist, *Proceedings-Separate No. 160*, ASCE, December, 1952.

described experiments involving pressures measured at various depths in an ice sheet. He found that the surface pressure reached a maximum a few hours after the start of the temperature rise, whereas the total pressure was not a maximum at that time because of the lag in the increase of temperature in the interior of the ice. Mr. Löfquist found that maximum total pressure occurs at a later time when the surface pressure is past its peak but the interior pressure is at a maximum. This results in a "pear-shaped" pressure curve.

An attempt was made by the writer to estimate the effect on a 2-ft thick ice sheet with a minimum surface temperature of $-30°$ F and to compare these results with those obtained by Mr. Monfore and Messrs. Brown and Clarke. For an air-temperature rise of 5° F, 10° F, and 15° F, respectively, the pressures obtained (using the relationships determined by Messrs. Brown and Clarke) would be 4,500 lb per lin ft, 5,000 lb per lin ft, and 7,000 lb per lin ft. Using the data derived by Mr. Monfore the pressures would be 20,000 lb per lin ft, 22,000 lb per lin ft, and 23,000 lb per lin ft.

These values are estimates, but wide differences are noted. These differences are caused partly by the variation in the structure and the variation in the arrangement of the ice crystals. In the field, ice structure and strength are bound to differ at various locations. It is likely that test procedure and test methods also are a factor.

It would be helpful if the air-temperature curve were plotted on Fig. 7 to show graphically the rapid increase in the air temperature required during the first fifteen minutes of the test. This would permit study of the lag in the ice-temperature rise relative to the air-temperature rise in these early stages and the influence on ice pressures.

The field observations made by Mr. Monfore regarding ice-pressure variations caused by the effect of restraint at the reservoir shores, the amount of snow cover, solar radiation, and wind conform to the expectations set forth in the writer's paper and elsewhere.

The wide variation of the basic data on the temperature-pressure relation as established by Mr. Monfore and Messrs. Brown and Clarke indicates the necessity for an explanation of these differences. Recognizing that the tests performed by the author were extensive, precise, and supported by field measurements, it is believed that verification is needed from independent sources—such as the test results from Sweden and Canada.[15,16] When a reasonably

[16] "Ice Pressure Against Dams: Some Investigations in Canada," by A. D. Hogg, *Proceedings-Separate No. 161*, ASCE, September, 1952.

consistent relationship has been established, then a revised set of design curves or tables should be prepared. Such design data should be in an easily adaptable form and should be in terms of variables that can be readily determined or accurately estimated. Generally, it would not be convenient to measure ice temperatures over a period of several years, as suggested by the author, but records of changes of air temperature and ice thicknesses usually would be available or could be estimated.

Until general agreement is reached on the basic properties affecting the development of ice pressure and these properties are supported by field measurements, the writer believes that the data presented in his 1947 paper, when used with judgment, can serve as a reasonable basis for estimating ice thrust. Before selection of a final value, some allowance should be made for the larger pressures indicated by Mr. Monfore.

Further field information is especially desirable, and since suitable gages have been developed, it is possible that they can be used under a wide variety of conditions. Measurement of pressures in smaller areas of ice sheets, such as in pumping-plant pits or in narrow channels where confinement is closer than in a large reservoir, would provide valuable data for use in the design of such structures.

G. E. MONFORE.[17]—In presenting the results of the USBR's experimental investigations of ice pressure it was recognized that much work remained to be done. When the program was discontinued in 1951, it was decided to make generally available the data which had been gathered up to that time. The results of the laboratory investigation were in agreement with the results obtained by Mr. Löfquist,[15] A detailed comparison is contained in a discussion of Mr. Löfquist's paper.[18]

[17] Research Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

[18] Discussion by G. E. Monfore of "Ice Pressure Against Dams: Studies of the Effects of Temperature Variations," by Bertil Löfquist, *Proceedings-Separate No. 374*, ASCE, December, 1953.

The pressure near the top of an ice sheet reaches a maximum before that of points lower in the sheet. Thrusts which were calculated from the pressures measured with electric pressure gages were based on the actual pressure gradient existing at a given time. These results are presented under the heading, "Field Investigations: Pressure Measurements." The method of using maximum pressures in computing thrusts on the basis of the laboratory tests and field ice temperatures was chosen because of its simplicity, and also because the results thus obtained were only slightly greater than the true values based on instantaneous gradients. For example, the greatest thrust measured at Eleven Mile Canon Reservoir on January 22, 1949, was 14 kips per lin ft. The thrust computed on the basis of maximum pressures without regard to time was 17 kips per lin ft. Similarly, the greatest measured thrust for January 20, 1949, was 16 kips per lin ft, and that computed on the basis of maximum pressures without regard to time was 17 kips per lin ft.

Although some of the earlier laboratory tests were continued considerably beyond the time required for the maximum pressure to develop, the pressure-time curve for ice can be given with certainty only to times which are approximately 25% greater than the time required for the maximum pressure to develop. It was found that the pressure-time curves, expressed in a dimensionless form, were nearly identical for the various initial temperatures and rates of temperature rise. The curve shown in Fig. 10 represents an average of many tests.

Temperatures were measured at several points in a laboratory ice sample for various air-temperature conditions in preliminary tests that have been described.[9] The air temperature and corresponding ice temperature for one

[9] "Laboratory Investigation of Ice Pressure," by G. E. Monfore, *Report No. SP-31*, Structural Research Lab., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., October 8, 1951.

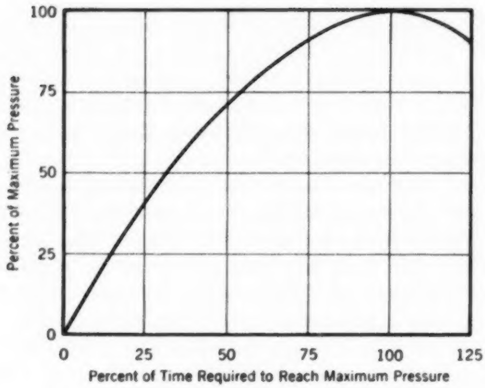


FIG. 10.—AVERAGE PRESSURE-TIME CURVE

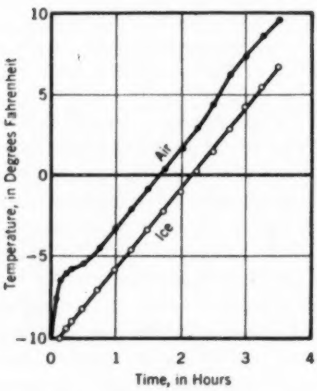


FIG. 11.—AIR AND ICE TEMPERATURES FOR TYPICAL LABORATORY TEST

typical test are shown in Fig. 11. This is the same test which is shown in Fig. 7. The rapid increase in air temperature for the first fifteen minutes was necessary to obtain a linear increase in ice temperature.

It is true that air temperatures are easier to measure in the field than are ice-sheet temperatures, but it is also true that the computation of ice temperatures from air temperatures is difficult. Surface conditions provide elements of uncertainty in such computations. The measurement of ice-sheet temperatures, compared to the measurement of ice pressures, is relatively simple; such data from various locations would be valuable, not only for direct estimation of thrust but also as a check on the methods of computing ice temperatures.

The presentation of data in an easily used form is desirable, but it must be remembered that no matter how convenient a table or set of curves may be, the accuracy of the presentation cannot be any greater than the accuracy of the original experimental data.

DISCUSSION OF RECENT ADDITIONS AND IMPROVEMENTS TO THE HALES BAR HYDROELECTRIC PLANT PROCEEDINGS-SEPARATE NO. 294

C. E. Blee, M. ASCE.—Mr. Meyer's paper, together with the presentation of today, gives a very interesting and very complete account of the recent additions to the Hales Bar hydro plant. An extensive alteration of an old power plant such as this always presents more problems, both from the standpoint of design and of construction, than would the building of a complete new plant of similar size. Also it offers greater opportunities to develop unique arrangements in design and ingenious methods in construction. Hales Bar has a particular interest since it is one of the pioneer hydroelectric plants of the country.

The paper tells of the foundation difficulties encountered in the construction of the original dam—how the time required and the cost greatly exceeded the estimates, and how it was finally necessary to resort to pneumatic caissons to get the foundations down to what appeared to be sufficiently sound rock. Then after completion, leakage developed under the dam which greatly impaired the usefulness and safety of the structure.

I do not believe that it could be said that the foundation rock at the Hales Bar site was worse than that at the Chickamauga Dam, 40 miles upstream, nor at the Fort Loudoun Dam. Both these dams were constructed by TVA employing open cofferdam methods but using extensive grouting and other devices for foundation treatment. The difference was, that 50 years ago they did not have the experience and the techniques which were developed later and which have made possible the utilization of sites which would have been considered impossible even 25 years ago.

Also they did not have the extensive subsurface exploration which we would have today so they did not know what they were up against. The hypothesis that disintegration of the rock would not occur below the permanent ground water level was a comforting one, but unfortunately not founded on facts. The cavitation of limestone rock below water level is not a process of erosion but is one of solution and requires only sufficient gradient to keep the water, which is in contact with the rock, constantly changing.

The failure of the asphalt grout to permanently stop the leakage was apparently not due to any deterioration of this material, but rather to the fact that the openings through the rock were not entirely filled and the high velocity leakage water gradually washed out the disintegrated rock, enlarging the water passages. During the subsequent drilling work, deposits of asphalt were frequently encountered by the drills and appeared sound and resilient.

In carrying out the additions, the construction forces developed a number of ingenious devices to overcome difficult situations and take advantage of repetitive operations. In lowering the elevation of the spillway crest, a slot was first cut in the concrete at each pier location just wide enough to accommodate the forming and pouring of the pier. In order to carry this work down below water level, a steel box-shaped caisson with rubber contact strips was fastened against the upstream face of the dam and pumped out. As soon as the concrete of the pier was brought up above water level, the caisson was moved to the next location. A great deal of line drilling and of drilling for blast holes and holes for anchor rods had to be done from the rollway of the dam. As

mentioned in Mr. Meyer's paper, a jumbo mounting the drifter drills was built on the job, equipped with adjustable, telescopic legs on the downstream side so that the working platform remained level as the jumbo was moved down the face of the dam. Floating equipment, such as cranes, barges and floating mixing plant, was used extensively.

The outstanding feature from a construction viewpoint, was the upstream cofferdam required to permit the removal of the non-overflow section of the dam next to the old powerhouse. This cofferdam was constructed of seventy-foot diameter steel sheet piling cells to withstand a normal head of 50 feet. I have learned to be careful about claiming any world records for the size of construction features and it was fortunate that we did not do so in this case for we learned later that two 76-foot diameter cells had been used at the Hadley Falls Station of the Holyoke Water-Power Company. However, for the conditions existing at the Hales Bar Site, it is my opinion that the height and diameter of the cofferdam cells were approaching the safe limit. The river bottom was a very irregular rock surface, affording no penetration for the piling. When the cofferdam was unwatered, exposing the downstream surface of the cells, the piling was found to be pulled out of its interlock in several places and the whole shell gave the general impression of being rather highly stressed. Steel plates were welded across the places where the interlocks were weakened and a low concrete kicking block, with a gutter to take care of leakage water, was poured against the toe of the shell and dowelled into the rock. All things considered, we were mighty glad to see the concrete, which replaced the dam section, get up above reservoir level. Any failure of the cofferdam while the main dam was breached, would, of course, be disastrous.

An interesting observation in connection with the addition of the new units is that although the 14 old units are decidedly obsolete and of very low efficiency, studies showed that the scrapping of these units could not be justified. They have a value for carrying peak loads of short duration and for operating during times when excess water is available, in all of which, efficiency is not of prime importance. It probably is true as a generalization that it is hard to justify the scrapping of hydroelectric equipment just as long as it will turn out power.

George R. Rich,[1] M. ASCE.—The author has presented an excellent account of the redevelopment of one of the pioneer multipurpose projects. From the historical standpoint the original construction affords an outstanding example of the need for adequate sub-surface exploration and expert interpretation of the results prior to design. In the light of present-day knowledge the earlier notion that solution channels could not cause damage to the foundation rock below the ground-water level appears curious but points a moral against the acceptance of theories unsubstantiated by actual exploration. If the speaker remembers correctly, the turbine waterways in the original substructure, (described in detail by Mr. Meyer) were designed by one of the most eminent hydraulic engineers of that time, Mr. Clemens Herschel, the inventor of the Venturi meter.

The striking feature of the overall design of the redevelopment is its cleanness; the power-station architecture is simple, functional and very effective. The design shown for the power-station substructure (Figure 12) is very efficient. In Kaplan turbine installations the ratio of wheel-diameter to head is always such that the combined mass of the intake and draft-tube may be utilized to sustain the hydrostatic loading from the reservoir. Because of the

1. Director, Chas. T. Main, Inc., Boston, Mass.

low entrance velocities the intake is properly comparatively short; and because of the necessity of regaining a high percentage of the energy in the velocity of exit from the turbine, the draft tube is relatively long. The structure is of necessity cellular in nature with the main piers constituting the principal elements of structural stability. These are made of liberal thickness, not only to facilitate final load delivery to the foundation rock, but also to provide adequate space for the workmen placing concrete around the reinforcement. The heavy roof slab of the scroll case serves as a horizontal girder to transmit the top reactions of the intermediate piers of the intake to the main piers. It will also be noted that the contraction joints in the main piers form a single unbroken plane through the entire cross-section.

On the basis of the cost data furnished by the author it is interesting to apply a few rough checks to the economic design of the redevelopment. The point of diminishing returns with respect to installed generating capacity is frequently in the vicinity of the 20 per cent of the regulated flow-duration curve. The Hales Bar Redevelopment with a turbine discharge 40,000 cfs appears to satisfy this criterion.

We are at liberty to check the economic justification for the power portion of the redevelopment by either of two bases. We may employ the values of energy and capacity and the corresponding fixed charges from the annual TVA report; or we may adopt the energy and capacity rates with the corresponding fixed charges for any representative private utility system. In the cases roughly tested by the speaker there appears to be a suitable margin of return on the investment by either method.

With respect to the investment in navigation facilities, we learn from the TVA annual reports that the TVA system between Paducah and Knoxville (a distance of some 600 miles) carries about 600 million ton-miles of freight annually at an estimated saving to the consumer of about $7,000,000 per year. Improvement of low-flow draft from 4 feet to 9 feet between Hales Bar and Chickamauga (a distance of about 7 miles) appears from the rough exploratory figures of the speaker to warrant the expenditure chargeable to navigation given in Mr. Meyer's tabulation.

DISCUSSION OF MOVEMENTS IN THE STRUCTURAL CONCRETE AT THE CONOWINGO HYDRO PLANT PROCEEDINGS-SEPARATE NO. 308

H. A. Kammer, A.M. ASCE.—The dilemma in which Messrs. Moyer and Hansen and their associates find themselves, in respect to the movements which have occurred in the structure of the Conowingo Hydro Plant, recalls all too clearly the similar situation that confronted engineers of the American Gas and Electric Company in the late 1930's and the early 1940's, when structural changes in the Buck Hydro Plant of the Appalachian Electric Power Company affected the operation of the three generators at that plant. Actually, the problem at Buck was considerably more severe than that at Conowingo, in view of the fact that certain of the machine components had suffered severe damage. Misalignment of the machines had become a serious operating problem before we were able to conclusively determine the nature of our troubles. Such conditions do not appear to exist at Conowingo, certainly not as of this time. The authors indicate that the movements which have occurred at Conowingo to date have, in no way, affected operating tolerances of the machines, the only operating problems being centered about the butterfly valves in the penstocks.

As in the case of Conowingo, photographic records indicate that the concrete used in the original installation at Buck was poured very wet. The coarse aggregate which was used was manufactured at the site, also in a manner similar to that at Conowingo. We hope, for the sake of our friends in the Philadelphia Electric Company, that the similarity ends there.

The concrete expansion in the Buck Plant substructure was traced to the native rock-phyllite—which was crushed and used as aggregate in the concrete. After exhaustive studies extending over several years, it was determined that the phyllite was reactive with certain ingredients in cement and that the end result was an expansion of the concrete. The detailed story of this phenomenon as it applied to the Buck Plant has been published in articles in both "Civil Engineering" and the "Journal of the American Concrete Institute."

It is sufficient here to simply say that the expansion of the concrete caused large cracks in the periphery of the pit liners of the units and in the guide vanes of the speed rings. Periodically, it was necessary to realign the machines in order to counteract the effect of the concrete growth. The width of the cracks in the pit liners was of the order of 1" at the time that the Buck Plant was shut down and portions of the substructure replaced. A new concrete foundation was installed for each of the three units, using low-alkali cement and sound aggregates and complying with modern-day techniques of concrete placing. This procedure has proven to be entirely successful to date. The machines have been operating successfully for the past ten years without any alignment troubles in spite of the fact that the surrounding concrete, which was not removed but was physically isolated from the new foundations, continues to grow.

As the authors point out, it is too early to permit the drawing of final conclusions as to the exact nature of the causes of movement at Conowingo. There appears to be some reason to suspect the presence of unsound aggregates which, in combination with the alkalies in the cement, are causing expansion

of the concrete. This appears to have been the case underlying the troubles that were noted in connection with the operation of the butterfly valves in the penstocks. However, it does not appear that this is the final answer on Conowingo. The elaborate system of controls which has been established should assist in ascertaining to a greater degree the exact nature of the movements which are occurring.

The authors mention the possibility of silt entrapment within cracks which may exist in the lower sections of the Conowingo concrete. However, no mention is made as to whether similar consideration has been given to the superstructure of the plant. It might be that a check of the condition of the superstructure would show the existence of some cracking into which foreign material has entered at some time during the thermal cycle so that, on contracting, those portions of the plant could not return to their original positions. It may be that certain expansion joints within the plant have been rendered ineffective for one reason or another, with the same results.

The fact that the movement at Conowingo appears to be horizontal to a much greater degree than vertical leads one to suspect that the major cause of this movement is not associated with the chemical growth of concrete. Rather, it appears that the trouble lies in some other cause, as yet undetermined, which results in this single directional movement. Of particular note seems to be the fact that the movement at the upper levels of the plant appears to be greater than that below the operating floor level. Could it be that some element of design or some factor associated with location and local conditions at the higher levels has caused the movement at these locations to be greater than that below ?

The authors have outlined the elaborate control system which has been instituted at Conowingo but it appears that no portion of this system relates to the north wall of the power house nor to any of the structures upstream therefrom. It would appear, therefore, that definite information can not be obtained directly as to whether the movements in the south wall result from changes within the power house proper or whether the movements of the south wall actually reflect movements which occur at or beyond the north wall.

The authors suggest the possibility of overall expansion of the concrete due to the annual thermal cycle. A corollary to this reasoning lies in the much shorter cycle of temperature changes which can occur as a result of the daily temperature cycle as well as the intermediate condition resulting from cyclical weather changes of relatively short duration. These conditions may be of particular concern in the roof slab which provides the base for the 220 kv switching station.

We are looking forward to receiving further reports on the investigations which are in process at Conowingo. We hope the authors and their associates are successful in isolating the causes of the movements at the hydro plant and that their next report will contain information both as to the causes and the remedies which were decided upon to alleviate the condition.

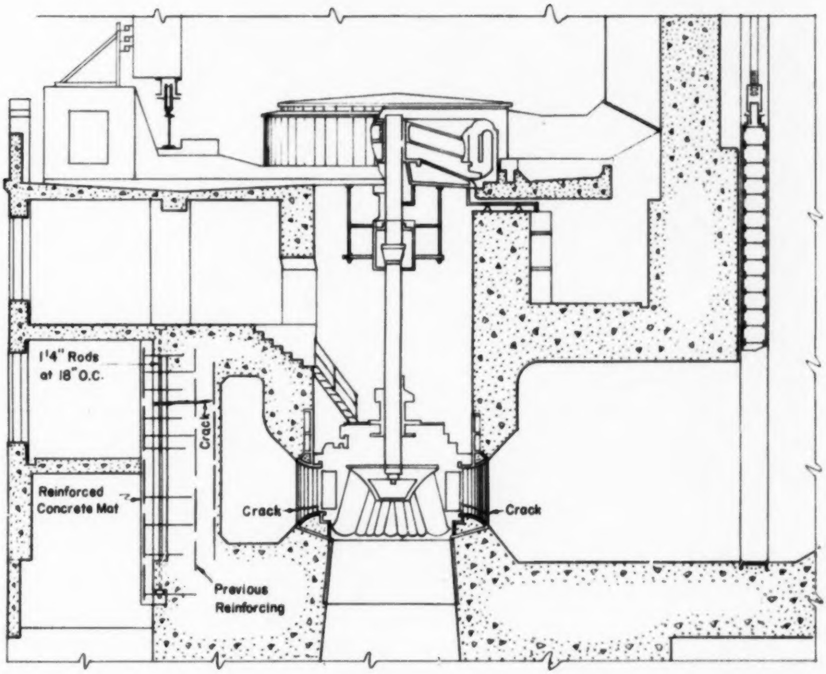
E. A. Woodhead.—The Idaho Power Company was made forcibly aware of the phenomenon of concrete growth about 20 years ago, when displacement of the concrete in the American Falls plant resulted in cracking of the downstream concrete walls of the semi-spiral casings and in cracking of the cast iron stay vanes of some of the turbines. Some misalignment of generators also occurred. Repairs were made to the turbine spiral casing wall by means of 1-1/4" vertical tie rods recessed into grooves cut in the face of the wall which were prestressed and then a well-reinforced and anchored facing slab was applied to the wall, as shown in Fig. 1. The turbine stay vanes were

repaired by means of plate steel brackets shaped to fit and studded to the cast iron, as shown in Fig. 2. The misaligned generators were re-set. These repairs have been satisfactory. A study of the causes which led to this above mentioned damage has guided our design and construction procedure in subsequently built plants. These causes and the remedies or deterrents applied are as follows:

1. Reactive Aggregates. Not much can be done about this except to select those aggregates which are found by test to be most desirable—or least undesirable.
2. High Alkali Cement. The cement most readily available in our recent construction work is somewhat alkaline. We have made a practice however of obtaining adequate supplies of a low alkali cement from a considerable distance to use in certain parts of the work where concrete growth would be liable to cause trouble.
3. Directional Expansion. Concrete growth will manifest itself ordinarily in the direction of least restraint, and in proportion to the length of the concrete prism involved in such expansion. This was the primary cause of deformation at American Falls, and we have guarded against it in subsequent plants by very heavy reinforcement of the turbine throat ring, and by heavy vertical reinforcement of the generator barrel, such reinforcement being of a cross sectional area proportional to the length of concrete prism involved. By this means such growth as might occur has been forced to take place in a direction which will cause the minimum of distortion of working clearances or settings. In the case of the Lower Salmon and Malad plants also, there was some doubt that low alkali cement would be available in sufficient quantity for our requirements, or at all, because of heavy governmental purchases of the most desirable cements, at that time, and the generators were therefore ordered with self-equalizing thrust bearings, as an additional precaution. We have actually been able however to use a low alkali cement (0.4 to 0.5% potassium and sodium oxide equivalent) in all power house substructure work around draft tube liners, throat rings and turbine blocks up to the generator foundation level.

In the C J Strike plant, which was completed early in 1952 we were able to make use of all the experience obtained up to that time. This plant is shown in Fig. 3. In addition to heavy ribbing on the throat ring and draft tube liner, and the use of the directional reinforcement previously mentioned, an innovation in the generator design was introduced, in conjunction with the generator manufacturer. Instead of the thrust bearing arms and the stator yoke being supported by concrete at different elevations, with the possibility of differential movement, the stator yoke is supported on a heavy steel ring which extends down to the elevation of the thrust bearing support, where the two elements mentioned are connected together as shown in Fig. 4. This has been so far entirely successful and the same idea was later followed at the Cabinet Gorge Plant.

As a means of checking possible vertical displacement of generators with respect to turbines due to concrete growth, we have made a practice for several years to provide a reference collar on the turbine shaft and appropriate reference marks on the turbine pit wall.



FIGURE—NO. 1

STEP NO 1 — Tie Rods and Dowels
STEP NO 2 — Reinforced Concrete Mat
and repair Stay Vanes

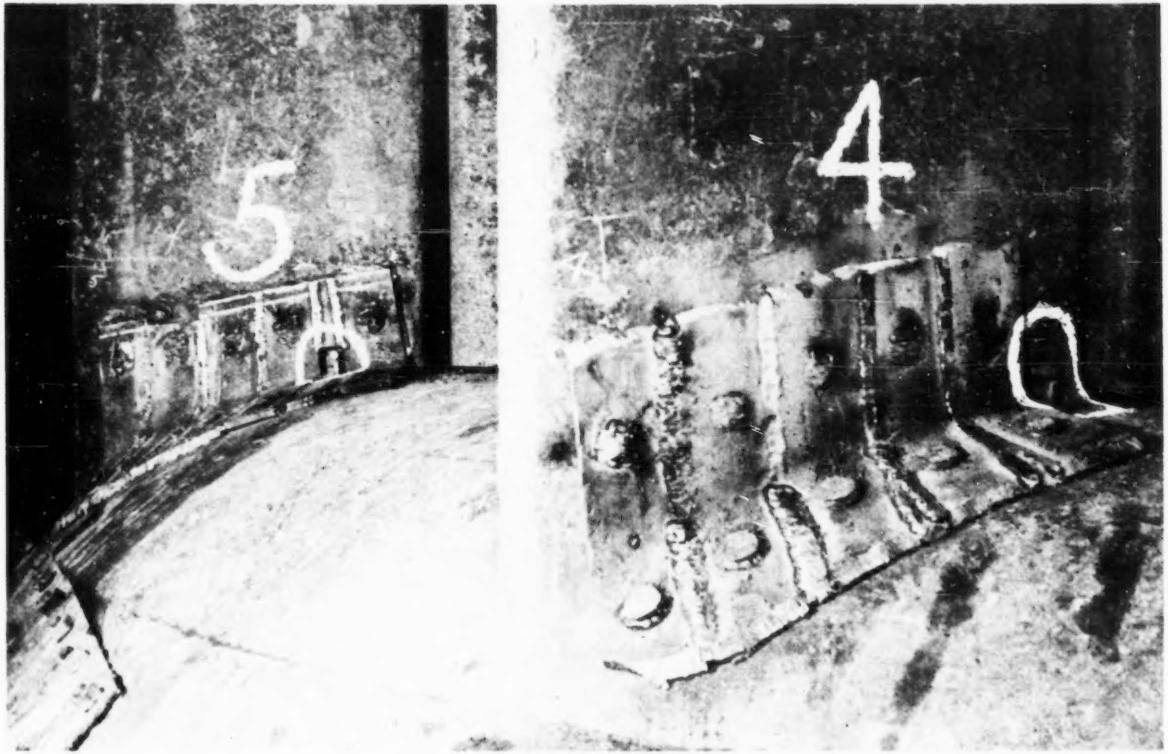


Fig. 2. Typical Repair of Stay Vanes

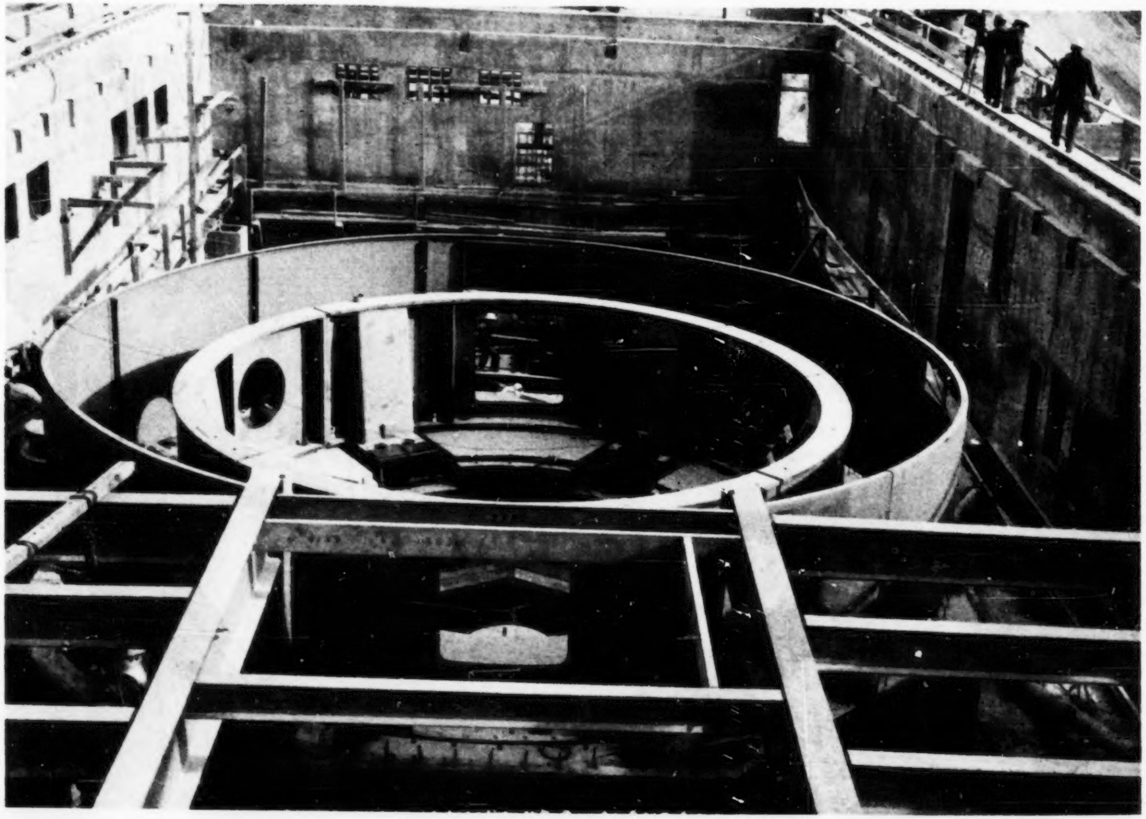
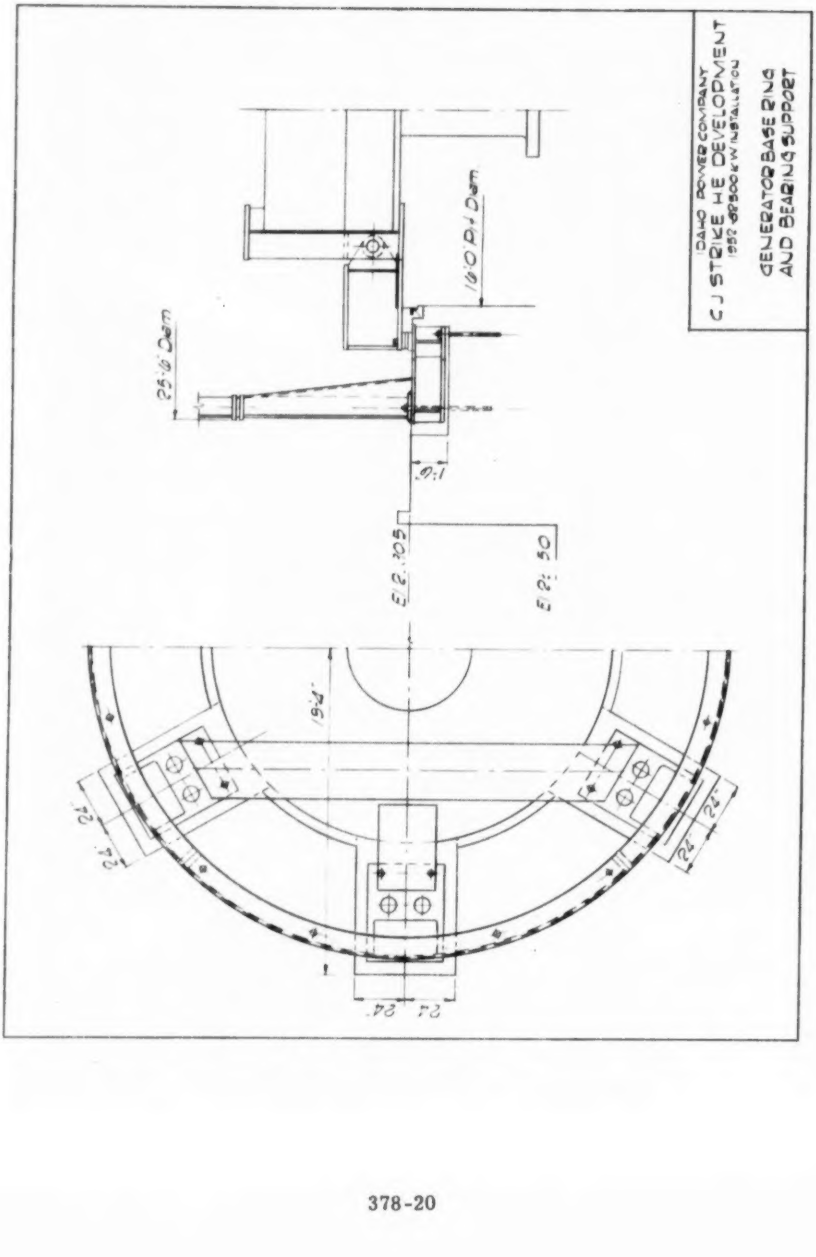


Fig. 3. Unit 2 Generator Base Ring and Lower Air Housing Roof Steel in Place Unit 1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a. Beginning with "Proceedings-Separate No. 200," published in July, 1953, the papers were printed by the photo-offset method.

b. Presented at the Miami Beach (Fla.) Convention of the Society in June, 1953.

c. Presented at the New York (N.Y.) Convention of the Society in October, 1953.

d. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.

e. Discussions, grouped by divisions.

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